Chapter 18*A* - Soils and Foundations

2001 CBC	PROPOSED ADOPTION	OSHPD		DSA-SS	Comments
		1	4		
	Adopt entire chapter without amendments				
	Adopt entire chapter with amendments listed below	X	X	X	
	Adopt only those sections listed below				
	1801A.1	X	X	X	
1801A.1.1 CA	*1801A.1.1 CA*	X	X	X	
	1801A.1.2 CA	X	X	X	
1804A.1 CA	*1802A.1*	X	X	X	Relocated existing California Building Standards into IBC format
	1802A.2	X	X	X	
	1802A.2.3	X	X	X	
	1802A.2.4	X	X	X	
	1802A.2.6	X	X	X	
	1802A.2.7	X	X	X	
1804A.3.8 CA	*1802A.2.8 CA*	X	X	X	Relocated existing California Building Standards into IBC format
1804A.2 CA	*1802A.4.1*	X	X	X	Relocated existing California Building Standards into IBC format
1637A CA	*1802A.6 CA*	X	X	X	Relocated existing California Building Standards into IBC format
1637A.2.1.1, 1804A.3, Item 6, 1804A.1	*1802A.7*	X	X	X	
1806A.4	*1805A.1*	X	X	X	Relocated existing California Building Standards into IBC format
1806A.2	*1805A.4.1*	X	X	X	Relocated existing California Building Standards into IBC format
Table 18A-I-C	*Table 1805A.4.2*	X	X	X	Relocated existing California Building Standards into IBC format
	1805A.4.2.3	X	X	X	
1806A.1 CA	*1805A.4.2.6*	X	X	X	Relocated existing California Building Standards into IBC format

	1805A.4.3	X	X	X	
	1805A.4.5	X	X	X	
	1805A.4.6	X	X	X	
1806A.11 CA	*1805A.4.7 CA*	X	X	X	Relocated existing California Building Standards into IBC format
	1805A.5	X	X	X	
	Tables 1805.5(1) through 1805.5 (5)				Stricken
	1805A.6	X	X	X	
	1805A.5.7.1	X	X	X	
1611A.6	*1806A.1*	X	X	X	Relocated existing California Building Standards into IBC format
1611A.13 CA	*1806A.2 CA*	X	X	X	Relocated existing California Building Standards into IBC format
	1807A.2	X	X	X	
	1808A.2.23.1	X	X	X	Editorial
	1808A.2.23.2 – Exceptions 2 and 3	X	X	X	
1806A.8.1	*1808A.2.23.2.4 CA*	X	X	X	Relocated existing California Building Standards into IBC format
	1809A.1	X	X	X	
	1809A.2.2.2.1	X	X	X	
	1809A.2.3.2.1	X	X	X	
	1809A.2.3.2.2	X	X	X	
	1810A.1.2.1	X	X	X	
	1810A.2	X	X	X	
	1810A.8.4.1	X	X	X	Editorial
	1811A.4	X	X	X	
	1812A.8	X	X	X	

REPEAL OF EXISTING CALIFORNIA AMENDMENTS IN PART OR IN WHOLE THAT ARE NO LONGER NECESSARY AS FOLLOWS:

2001 CBC DIVISION I – GENERAL

~~2001 CBC SECTION 1802A – QUALITY AND DESIGN:~~ Repeal all amendments in this section.

2001 CBC SECTION 1804A – FOUNDATION INVESTIGATION: Repeal amendments in following subsections.

~~1804A.1, 1804A.3~~ and ~~1804A.4~~.

2001 CBC SECTION 1806A – FOOTINGS: Repeal amendment in the following section.

~~1806A.1, 1806A.3, 1806A.6~~ including all subsections.

~~2001 CBC SECTION 1807A – PILES – GENERAL REQUIREMENTS:~~ Repeal all amendments in this section including all subsections.

~~2001 CBC SECTION 1808A – SPECIFIC PILES REQUIREMENTS:~~ Repeal all amendments in this section including all subsections.

2001 CBC SECTION 1809A – FOUNDATION CONSTRUCTION – SEISMIC ZONES 3 & 4: Repeal amendments in following subsections.

~~1809A.5.1~~ and ~~1809A.5.2.1~~.

EXPRESS TERMS

SECTION 1801*A* - GENERAL

1801*A*.1 Scope. The provisions of this chapter shall apply to building and foundation systems in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with Chapter 16*A*.

(Relocated from 1801A.1.1, CBC 2001) Refer to Appendix J: Grading, for requirements governing grading, excavation and earthwork construction, including fills and embankments.

***1801A.1.1 Application.** The scope of application of Chapter 18A is as follows:*

1. Applications listed in Section 109.2 regulated by the Division of the State Architect-Structural Safety (DSA-SS).These applications include public elementary and secondary schools, community colleges and state-owned or state-leased essential services buildings.

2. Applications listed in Section 110.1, and 110.4 regulated by the Office of Statewide Health Planning and Development (OSHPD).These applications include hospitals, skilled nursing facilities, intermediate care facilities and correctional treatment centers.

*__Exception__ [For OSHPD 2]**:** Single-story Type V skilled nursing or intermediate care facilities utilizing wood-frame or light-steel-frame construction as defined in Health and Safety Code Section 129725, which shall comply with CBC Chapter 18 and any applicable amendments therein.*

***1801.1.2 Amendments in this chapter.** DSA - SS and OSHPD adopt this chapter and all amendments.*

***Exception:** Amendments adopted by only one agency appear in this chapter preceded with the appropriate acronym of the adopting agency, as follows:*

***1.** Division of the State Architect - Structural Safety:*
***[DSA-SS] -** For applications listed in Section 109.2*

***2.** Office of Statewide Health Planning and Development:*
***[OSHPD 1]** - For applications listed in Section 110.1*
***[OSHPD 4]** - For applications listed in Section 110.4*

1801*A*.2 Design. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605*A*.3. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in Chapters 16*A*, 19*A*, 21*A*, 22*A* and 23 of this code. Excavations and fills shall also comply with Chapter 33.

1801*A*.2.1 Foundation design for seismic overturning. Where the foundation is proportioned using the load combinations of Section 1605*A*.2, and the computation of the seismic overturning moment is by the equivalent lateral-force method or the modal analysis method, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.

SECTION 1802*A* - FOUNDATION AND SOILS INVESTIGATIONS

1802*A*.1 General. Foundation and soils investigations shall be conducted in conformance with Sections 1802*A*.2 through ~~1802.7~~*1802A.8*. ~~Where required by the building official, the classification and investigation of the soil shall be made by a registered design professional.~~ *(Relocated from 1804A.1, CBC 2001)The classification and investigation of the soil shall be made under the responsible charge of a California registered geotechnical engineer. All recommendations contained in geotechnical and engineering geology reports shall be subject to the approval of the enforcement agency, in consultation with the ~~California Division of Mines and Geology (DMG) /~~ California Geological Survey (CGS). All reports shall be prepared and signed by a registered geotechnical engineer and an engineering geologist where applicable.*

1802*A*.2 Where required. The owner or applicant shall submit a foundation and soils investigation to the building official where required in Sections 1802*A*.2.1 through ~~1802.2.7~~ *1802A.2.8*.

Exception: ~~The building official need not require a foundation or soils investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 1802.2.1 through 1802.2.6.~~ *Geotechnical reports are not required for one-story, wood-frame and light-steel-frame buildings of Type II or Type V construction and 4,000 square feet (371m^2) or less in floor area, not located within Earthquake Fault Zones or Seismic Hazard Zones as shown in the most recently published maps from the ~~California Division of Mines and Geology (DMG) /~~ California Geological Survey (CGS). Allowable foundation and lateral soil pressure values may be determined from Table 1804A.2.*

1802*A*.2.1 Questionable soil. Where the classification, strength or compressibility of the soil are in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 1802*A*.4 through ~~1802.6~~ *1802A.7.*

1802*A*.2.2 Expansive soils. In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist.

1802*A*.2.3 Ground-water table. A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

Exception: *Not permitted by OSHPD and DSA-SS*. ~~A subsurface soil investigation shall not be required where waterproofing is provided in accordance with Section 1807.~~

1802*A*.2.4 Pile and pier foundations. Pile and pier foundations shall be designed and installed on the basis of a foundation investigation and report as specified in Sections 1802*A*.4 through ~~1802.6~~ *1802A.7* and Section 1808*A*.2.2.

1802*A*.2.5 Rock strata. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

1802*A*.2.6 Seismic Design Category C. Where a structure is determined to be in Seismic Design Category C ~~in accordance with Section 1613~~, an investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

1802*A*.2.7 Seismic Design Category D, E or F. Where the structure is determined to be in Seismic Design Category D, E or F, in accordance with Section 1613*A*, the soils investigation requirements for Seismic Design Category C, given in Section 1802*A*.2.6, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.

2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include but are not limited to ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7.

Exception: A site-specific study need not be performed, provided that peak ground acceleration equal to $S_{DS}/2.5$ is used, where S_{DS} is determined in accordance with Section ~~21.2.1 of ASCE 7~~ *1613A*.

1802A.2.8 *(Relocated from 1804A.3, Item 8, CBC 2001)* ***High Sulfate Soils.*** *In areas subject to high sulfate soils, an evaluation of the impact on the durability of concrete foundations shall be considered.*

1802*A*.3 Soil classification. Where required, soils shall be classified in accordance with Section 1802*A*.3.1 or 1802*A*.3.2.

1802*A*.3.1 General. For the purposes of this chapter, the definition and classification of soil materials for use in Table 1804*A*.2 shall be in accordance with ASTM D 2487.

1802*A*.3.2 Expansive soils. Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.

2. More than 10 percent of the soil particles pass a No. 200 sieve (75µm), determined in accordance with ASTM D 422.

3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.

4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

1802*A*.4 Investigation. Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

1802*A*.4.1 Exploratory boring. The scope of the soil investigation including the number and types of borings or soundings, the equipment used to drill and sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional.

(Relocated from 1804A.2, CBC 2001) *~~Whenever it is necessary to make special investigations, sufficient borings or exploration shafts shall be made as deemed necessary by the geotechnical engineer to evaluate the character of the soil under the entire building or structure, except that there~~* *There shall not be less than one boring or exploration shaft for each 5,000 square feet (465 m^2) of building area at the foundation level with a minimum of two provided for any one building.* *~~The possibility of liquefaction under seismic disturbance shall be considered in the investigation. If there is a potential for liquefaction, the geotechnical engineer shall report the estimated amount of displacement.~~* *A boring may be considered to reflect subsurface conditions relevant to more than one building, subject to the approval of the enforcement agency.*

Borings shall be of sufficient size to permit visual examination of the soil in place or, in lieu thereof, cores shall be taken.

Borings shall be of sufficient depth and size to adequately characterize sub-surface conditions.

1802*A*.5 Soil boring and sampling. The soil boring and sampling procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring and sampling operations.

1802A.6 *(Relocated from 1637A, CBC 2001)* ***Site data*** *~~FOR HOSPITALS AND STATE-OWNED OR STATE-LEASED ESSENTIAL SERVICES BUILDINGS.~~*

1802A.6.1 *(Relocated from 1637A.1, CBC 2001)* ***Engineering geologic reports.***

1802A.6.1.1 *Geologic and earthquake engineering reports shall be required for all proposed construction.*

EXCEPTIONS:

1. *Reports are not required for one-story, wood-frame and light-steel-frame buildings of Type II or Type V construction and 4,000 square feet (371m^2) or less in floor area, not located within Earthquake Fault Zones or Seismic Hazard Zones as shown in the most recently published maps from the ~~California Division of Mines and Geology (DMG) /~~ California Geological Survey (CGS); nonstructural, associated structural or nonrequired structural alterations and incidental structural additions or alterations, and structural repairs for other than earthquake damage.*

2. *A previous report for a specific site may be resubmitted, provided that a reevaluation is made and the report is found to be currently appropriate.*

1802A.6.1.2 *(Relocated from 1637A.1.2, CBC 2001) The purpose of the engineering geologic report shall be to identify geologic and seismic conditions that may require project mitigations. The reports shall contain data which provide an assessment of the nature of the site and potential for earthquake damage based on appropriate investigations of the regional and site geology, project foundation conditions and the potential seismic shaking at the site. The report shall be prepared by a California-certified engineering geologist in consultation with a California-registered geotechnical engineer. ~~The engineering geologic report shall not contain design criteria, but shall contain basic data to be used for a preliminary earthquake engineering evaluation of the project.~~*

The preparation of the engineering geologic report shall consider the most recent ~~Division of Mines and Geology~~ ~~DMG /~~ CGS ~~Notes 44 and 42~~ Note 48: Checklist for the Review of Engineering Geology and Seismology Reports for California Public School, Hospitals, and Essential Services Buildings. ~~Guidelines for reparing Engineering Geologic Reports, and Guidelines to Geologic/Seismic Reports, respectively. Upper-bound earthquakes, proposed in the Engineering Geologic Report, must be fully supported by satisfactory data and analysis.~~ In addition, the most recent version of ~~DMG /~~ CGS Special Publication 42, Fault Rupture Hazard Zones in California, shall be considered for project sites proposed within an Alquist-Priolo ~~special studies zone~~ Earthquake Fault Zone. The most recent version of ~~DMG /~~ CGS Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California, shall be considered for project sites proposed within a Seismic Hazard Zone. All conclusions shall be fully supported by satisfactory data and analysis.

The report shall include, but shall not be limited to, the following:

1. *Geologic investigation.*

2. *Evaluation of the known active and potentially active faults, both regional and local. ~~including estimates of their upper-bound earthquakes and estimates of the peak ground accelerations at the site resulting from these earthquakes.~~*

3. *Ground-motion parameters, as required by Section 1613A, 1614A & ASCE 7.*

~~3~~ .4. *Evaluation of slope stability at or near the site, ~~and the liquefaction and settlement potential of the earth materials in the foundation~~.*

5. *The liquefaction and settlement potential of the earth materials in the foundation.*

~~*1637A.1.3 The engineering geologic report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic problems and hazards are adequately identified and described in order to provide a timely completion of the subsequent geotechnical report, described in Section 1637A.2.1. The enforcement agency, with consultation of its advisors, may require additional information, analysis and/or clarification of potential geologic problems affecting the pro posed building site before approval is given. The results of the approved engineering geologic report shall be used as a basis for further investigations for the geotechnical report. Approval of the engineering geologic report by the enforcement agency shall be required prior to the submission of the geotechnical report.*~~

~~*1637A.2 Geotechnical and Supplemental Ground-response Reports.*~~

~~*1637A.2.1.2 The geotechnical report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic hazards and foundation problems have been adequately evaluated. The enforcement agency, with the consultation of its advisors, may require additional information, analysis or clarification of potential geotechnical issues affecting the proposed building site before approving the geotechnical report. Approval of the geotechnical report by the enforcement agency shall be required prior to the approval of the supplemental ground-response report, if required, as described in Section 1637A.2.2. The results of the geotechnical report shall be used as a guide for further investigations for the supplemental ground-response report.*~~

1802A.6.2 *(Relocated from 1637A.2.2, CBC 2001)* ***Supplemental ground-response report.*** *If site-specific ground-motion procedures, as set forth in ASCE 7 Chapter 21, or ground-motion time-history analysis, as set forth in ASCE 7 Chapter 16, Section 17.3 or Section 18.2.3,are used for design, then a supplemental ground-response report may be required. All conclusions and ground-motion parameters shall be fully supported by satisfactory data and analysis.*

~~*1637A.2.2 Supplemental ground-response report. A supplemental ground-response report may be required, containing aground-motion element and an advanced geotechnical element.*~~

1802A.6.2.1 *(Relocated from 1637A.2.2.1, CBC 2001) The ground-motion element shall be prepared by a registered ~~civil~~ geotechnical engineer or geophysicist (depending on the scope of the element), or engineering geologist licensed in the state of California, and having professional specialization in earthquake analyses. The ground-motion element shall present a detailed characterization of earthquake ground motions for the site, which incorporates data given in the geotechnical report. The level of ground motion considered by the ground-motion element shall be as described in ~~Section 1631A.2~~ ASCE 7 Chapter 21. The characterization of ground motion in the ground-motion element shall be given, according to the requirements of the analysis, in terms of:*

~~*1. Peak acceleration, bracketed duration and predominant period.*~~

~~2.~~ 1. *Elastic structural response spectra.*

~~3.~~ 2. *Time-history plot of predicted ground motion at the site.*

~~4.~~ 3. *Other analyses in conformance with accepted engineering and seismological practice.*

1802A.6.2.2 *(Relocated from 1637A.2.2.2, CBC 2001) The advanced geotechnical element shall contain the results of dynamic geotechnical analyses specified by the approved geotechnical report. Where site response analysis, as set forth in ASCE 7 Section 21.1, is required, the response model shall be fully explained. The input data and assumptions shall be fully documented, and the surface ground motions recommended for design shall be clearly identified.*

The supplemental ground-response report shall be submitted to the enforcement agency for review and approval. The review shall determine whether the ground-motion response evaluations of the site are adequately represented. The enforcement agency, ~~under~~ after consultation with its advisors, may require additional information, analysis or clarification of potential ground-response issues reported in the supplemental ground-response report for the proposed building site.

~~1802. 6~~ *1802A. 7 Geotechnical* Reports. The soil classification and design load-bearing capacity shall be shown on the construction document. Where required by the building official, a written report of the investigation shall be submitted ~~that includes~~. *(Relocated from 1637A.2.1.1, 2001 CBC) The geotechnical report shall provide completed evaluations of the foundation conditions of the site and the potential geologic / seismic hazards affecting the site. The geotechnical report shall include, but shall not be limited to, site-specific evaluations of design criteria related to the nature and extent of foundation materials, groundwater conditions, liquefaction potential, settlement potential and slope stability. The report shall contain the results of the analyses of problem areas identified in the engineering geologic report. The geotechnical report shall incorporate estimates of the characteristics of site ground motion provided in the engineering geologic report.* ~~*The estimates of ground motion shall not be structural design criteria, but shall be provided to characterize the seismic environment of the site, with consideration of the upper-bound earthquakes reported in the engineering geologic report. The ground-motion estimates shall include, but shall not be limited to, peak ground motions and predominant period. The estimates should be derived by accepted methods of current seismological practice and fully documented in the geologic report.*~~

~~*The geotechnical report shall be prepared by a geotechnical engineer registered in the state of California with the advice of the certified engineering geologist and other technical experts, as necessary. The approved engineering geologic report shall be submitted with or as part of the geotechnical report.*~~ *The geotechnical report shall include,* but need not be limited to, the following information:

1. A plot showing the location of test borings and/or excavations.
2. A complete record of the soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered. *Historic high ground water elevations shall be addressed in the report to adequately evaluate liquefaction and settlement potential.*
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
6. Expected total and differential settlement.
7. Pile and pier foundation information in accordance with Section 1808*A*.2.2.
8. Special design and construction provisions for footings or foundations founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section 1803*A*.5.
10. *(Relocated from 1804A.3 Item 6, CBC 2001) The report shall consider the effects of stepped footings addressed in Section 1805A.1.*
11. *(Relocated from 1804A.1, CBC 2001) The report shall consider the effects of seismic hazards per Section 1802A.6.*

SECTION 1803*A* - EXCAVATION, GRADING AND FILL

1803*A*.1 Excavations near footings or foundations. Excavations for any purpose shall not remove lateral support from any footing or foundation without first underpinning or protecting the footing or foundation against settlement or lateral translation.

1803*A*.2 Placement of backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or dampproofing material.

Exception: Controlled low-strength material need not be compacted.

1803*A*.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation is permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

1803*A*.4 Grading and fill in flood hazard areas. In flood hazard areas established in Section 1612*A*.3, grading and / or fill shall not be approved:

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.

2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a registered design professional in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the design flood.

3. In flood hazard areas subject to high-velocity wave action, unless such fill is conducted and / or placed to avoid diversion of water and waves toward any building or structure.

4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed flood hazard area encroachment, when combined with all other existing and anticipated flood hazard area encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

1803*A*.5 Compacted fill material. Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.

2. Specifications for material to be used as compacted fill.

3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.

4. Maximum allowable thickness of each lift of compacted fill material.

5. Field test method for determining the in-place dry density of the compacted fill.

6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.

7. Number and frequency of field tests required to determine compliance with Item 6.

Exception: Compacted fill material less than 12 inches (305 mm) in depth need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

1803*A*.6 Controlled low-strength material (CLSM). Where footings will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

SECTION 1804*A* - ALLOWABLE LOAD-BEARING VALUES OF SOILS

1804*A*.1 Design. The presumptive load-bearing values provided in Table 1804*A*.2 shall be used with the allowable stress design load combinations specified in Section 1605*A*.3.

1804*A*.2 Presumptive load-bearing values. The maximum allowable foundation pressure, lateral pressure or lateral sliding-resistance values for supporting soils near the surface shall not exceed the values specified in Table 1804*A*.2 unless data to substantiate the use of a higher value are submitted and approved.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions.

Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity is permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

TABLE 1804*A*.2 - ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS	ALLOWABLE FOUNDATION PRESSURE (psf)[d]	LATERAL BEARING (psf/f below natural grade)[d]	LATERAL SLIDING	
			Coefficient of friction[a]	Resistance (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500[c]	100	—	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.

a. Coefficient to be multiplied by the dead load.

b. Lateral sliding resistance value to be multiplied by the contact area, as limited by Section 1804*A*.3.

c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation.

d. An increase of one-third is permitted when using the alternate load combinations in Section 1605*A*.3.2 that include wind or earthquake loads.

1804*A*.3 Lateral sliding resistance. The resistance of structural walls to lateral sliding shall be calculated by combining the values derived from the lateral bearing and the lateral sliding resistance shown in Table 1804*A*.2 unless data to substantiate the use of higher values are submitted for approval.

For clay, sandy clay, silty clay and clayey silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

1804*A*.3.1 Increases in allowable lateral sliding resistance. The resistance values derived from the table are permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 0.5 inch (12.7 mm) motion at the ground surface due to short-term lateral loads are permitted to be designed using lateral-bearing values equal to two times the tabular values.

SECTION 1805*A* - FOOTINGS AND FOUNDATIONS

1805*A*.1 General. Footings and foundations shall be designed and constructed in accordance with Sections 1805*A*.1 through 1805*A*.9. Footings and foundations shall be built on undisturbed soil, compacted fill material or CLSM. Compacted fill material shall be placed in accordance with Section 1803*A*.5. CLSM shall be placed in accordance with Section 1803*A*.6.

The top surface of footings shall be level. The bottom surface of footings is permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

(Relocated from 1806A.4, CBC 2001) Individual steps in continuous footings shall not exceed 18 inches (457 mm) in height and the slope of a series of such steps shall not exceed 1 unit vertical to 2 units horizontal (50% slope) unless otherwise recommended by a soils report. The steps shall be detailed on the drawings. The local effects due to the discontinuity of the steps shall be considered in the design of the foundation.

1805*A*.2 Depth of footings. The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the depth of footings shall also conform to Sections 1805*A*.2.1 through 1805*A*.2.3.

1805*A*.2.1 Frost protection. Except where otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures shall be protected by one or more of the following methods:

1. Extending below the frost line of the locality;

2. Constructing in accordance with ASCE 32; or

3. Erecting on solid rock.

Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. Classified in Occupancy Category I, in accordance with Section 1604*A*.5;

2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and

3. Eave height of 10 feet (3048 mm) or less.

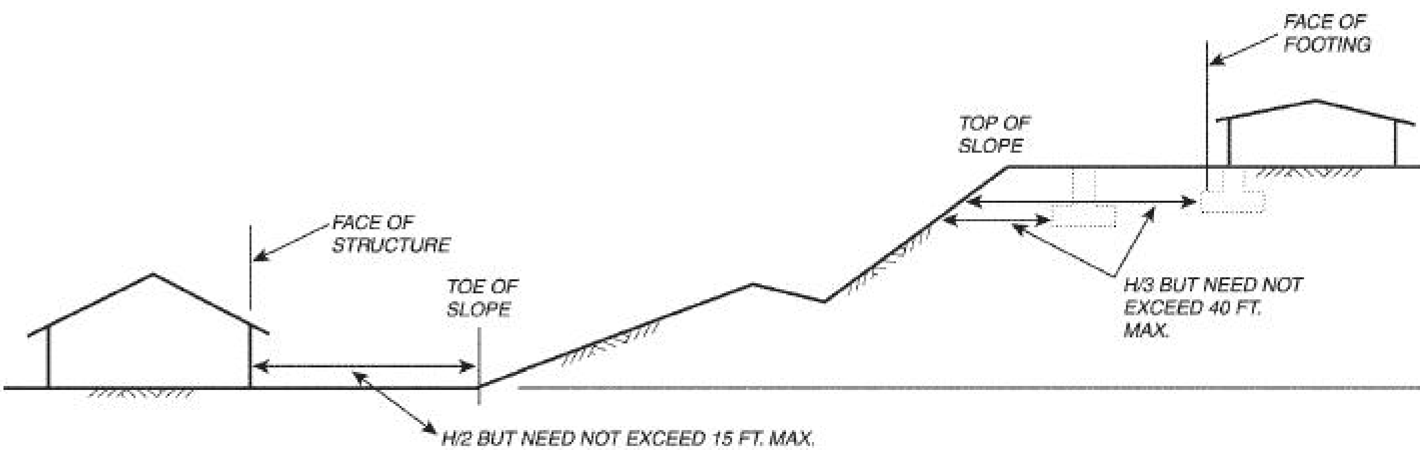
Footings shall not bear on frozen soil unless such frozen condition is of a permanent character.

1805*A*.2.2 Isolated footings. Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

1805*A*.2.3 Shifting or moving soils. Where it is known that the shallow subsoils are of a shifting or moving character, footings shall be carried to a sufficient depth to ensure stability.

1805*A*.3 Footings on or adjacent to slopes. The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall conform to Sections 1805*A*.3.1 through 1805*A*.3.5.

1805*A*.3.1 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Section 1805*A*.3.5 and Figure 1805*A*.3.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.



For SI: 1 foot = 304.8 mm.

FIGURE 1805*A*.3.1 - FOUNDATION CLEARANCES FROM SLOPES

1805*A*.3.2 Footing setback from descending slope surface. Footings on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Section 1805A.3.5 and Figure 1805A.3.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

1805A.3.3 Pools. The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

1805A.3.4 Foundation elevation. On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

1805A.3.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

1805A.4 Footings. Footings shall be designed and constructed in accordance with Sections 1805A.4.1 through 1805A.4.6.

1805A.4.1 Design. Footings shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. The minimum width of footings shall be 12 inches (305 mm).

Footings in areas with expansive soils shall be designed in accordance with the provisions of Section 1805A.8.

(Relocated from 1806A.2, CBC 2001) The enforcing agency may require an ~~elastic~~ analysis at footing and grade beam elements to determine subgrade deformations in order to evaluate their effect on the superstructure drift values in Chapter 16A.

1805A.4.1.1 Design loads. Footings shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605A.2 or 1605A.3. The dead load is permitted to include the weight of foundations, footings and overlying fill. Reduced live loads, as specified in Sections 1607A.9 and 1607A.11, are permitted to be used in the design of footings.

1805A.4.1.2 Vibratory loads. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the footing design to prevent detrimental disturbances of the soil.

1805A.4.2 Concrete footings. The design, materials and construction of concrete footings shall comply with Sections 1805A.4.2.1 through 1805A.4.2.6 and the provisions of Chapter 19A.

Exception: Where a specific design is not provided, concrete footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805A.4.2.

TABLE 1805A.4.2 - FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION[a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING[f]	WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)
1	12	6
2	15	6
3	18	8[g]

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

a. Depth of footings shall be in accordance with Section 1805*A*.2.

b. The ground under the floor is permitted to be excavated to the elevation of the top of the footing.

c. *(Relocated from Table 18A-I-C, CBC 2001)* Not permitted by OSHPD and DSA-SS. ~~Interior stud bearing walls are permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.~~

d. See Section 1908*A* for additional requirements for footings of structures assigned to Seismic Design Category ~~C,~~ D, E or F.

e. For thickness of foundation walls, see Section 1805*A*.5.

f. Footings are permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.

g. Plain concrete footings for Group R-3 occupancies are permitted to be 6 inches thick.

1805*A*.4.2.1 Concrete strength. Concrete in footings shall have a specified compressive strength (f'_c) of not less than 2,500 pounds per square inch (psi) (17 237 kPa) at 28 days.

1805*A*.4.2.2 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613*A*, individual spread footings founded on soil defined in Section 1613*A*.5.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient, S_{DS}, divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

1805*A*.4.2.3 Plain concrete footings. *Not permitted by OSHPD and DSA-SS.* ~~The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.~~

> ~~**Exception:** For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.~~

1805*A*.4.2.4 Placement of concrete. Concrete footings shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water.

1805*A*.4.2.5 Protection of concrete. Concrete footings shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

1805*A*.4.2.6 Forming of concrete. Concrete footings are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require forming. Where forming is required, it shall be in accordance with Chapter 6 of ACI 318.

(Relocated from 1806A.1, CBC 2001) *The horizontal dimensions of unformed concrete footings shall be increased 1 inch (25 mm) at every vertical surface at which concrete is placed directly against the soil.*

1805*A*.4.3 Masonry-unit footings. *Not permitted by OSHPD and DSA-SS.* ~~The design, materials and construction of masonry-unit footings shall comply with Sections 1805.4.3.1 and 1805.4.3.2, and the provisions of Chapter 21~~.

> ~~Exception: Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction are permitted to be designed in accordance with Table 1805.4.2.~~

~~1805.4.3.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.8 and the depth shall not be less than twice the projection beyond the wall, pier or column. The width shall not be less than 8 inches (203 mm) wider than the wall supported thereon.~~

~~1805.4.3.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be 1.5 inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.~~

1805*A*.4.4 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

1805*A*.4.5 Timber footings. *Not permitted by OSHPD and DSA-SS.* ~~Timber footings are permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to the grain in untreated timber footings supported upon treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AF&PA NDS.~~

1805*A*.4.6 Wood foundations. *Not permitted by OSHPD and DSA-SS.* ~~Wood foundation systems shall be designed and installed in accordance with AF&PA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.~~

1805A.4.7 *(Relocated from 1806A.11, CBC 2001)* ***Pipes and Trenches.*** *Unless otherwise recommended by the soils report, open or backfilled trenches parallel with a footing shall not be below a plane having a downward slope of 1 unit vertical to 2 units horizontal (50% slope) from a line 9 inches (229 mm) above the bottom edge of the footing, and not closer than 18 inches (457 mm) from the face of such footing.*

Where pipes cross under footings, the footings shall be specially designed. Pipe sleeves shall be provided where pipes cross through footings or footing walls and sleeve clearances shall provide for possible footing settlement, but not less than 1 inch (25 mm) all around pipe.

1805*A*.5 Foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19*A* or 21*A*, respectively. ~~Foundation walls that are laterally supported at the top and bottom and within the parameters of Tables 1805.5(1) through 1805.5(5) are permitted to be designed and constructed in accordance with Sections 1805.5.1 through 1805.5.5.~~

~~**TABLE 1805.5(1) - PLAIN MASONRY FOUNDATION WALLS[a, b, c]**~~

~~MAXIMUM WALL HEIGHT (feet)~~	~~MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)~~	~~MINIMUM NOMINAL WALL THICKNESS (inches)~~ ~~Soil classes and lateral soil load[a] (psf per foot below natural grade)~~ ~~GW, GP, SW and SP soils 30~~	~~GM, GC, SM, SM-SC and ML soils 45~~	~~SC, ML-CL and Inorganic CL soils 60~~
~~7~~	~~4 (or less)~~	~~8~~	~~8~~	~~8~~
	~~5~~	~~8~~	~~10~~	~~10~~
	~~6~~	~~10~~	~~12~~	~~10 (solid[e])~~
	~~7~~	~~12~~	~~10 (solid[e])~~	~~10 (solid[e])~~
~~8~~	~~4 (or less)~~	~~8~~	~~8~~	~~8~~
	~~5~~	~~8~~	~~10~~	~~12~~
	~~6~~	~~10~~	~~12~~	~~12 (solid[e])~~
	~~7~~	~~12~~	~~12 (solid[e])~~	~~Note d~~
	~~8~~	~~10 (solid[e])~~	~~12 (solid[e])~~	~~Note d~~
~~9~~	~~4 (or less)~~	~~8~~	~~8~~	~~8~~
	~~5~~	~~8~~	~~10~~	~~12~~

	~~6~~ ~~7~~ ~~8~~ ~~9~~	~~12~~ ~~12 (solid[e])~~ ~~12 (solid[e])~~ ~~Note d~~	~~12~~ ~~12 (solid[e])~~ ~~Note d~~ ~~Note d~~	~~12 (solid[e])~~ ~~Note d~~ ~~Note d~~ ~~Note d~~

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.~~

~~a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

~~b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.2.~~

~~c. Solid grouted hollow units or solid masonry units.~~

~~d. A design in compliance with Chapter 21 or reinforcement in accordance with Table 1805.5(2) is required.~~

~~e. For height of unbalanced backfill, see Section 1805.5.1.2.~~

~~TABLE 1805.5(2) - 8-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d^{3} 5 INCHES[a, b, c]~~

~~MAXIMUM WALL HEIGHT (feet-inches)~~	~~MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)~~	~~VERTICAL REINFORCEMENT~~ ~~Soil classes and lateral soil load[a] (psf per foot below natural grade)~~		
		~~GW, GP, SW and SP soils 30~~	~~GM, GC, SM, SM-SC and ML soils 45~~	~~SC, ML-CL and Inorganic CL soils 60~~
~~7-4~~	~~4-0 (or less)~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~5-0~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~6-0~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~	~~#5 at 48″ o.c.~~
	~~7-4~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~
~~8-0~~	~~4-0 (or less)~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~5-0~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~6-0~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~	~~#5 at 48″ o.c.~~
	~~7-0~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~
	~~8-0~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~
~~8-8~~	~~4-0 (or less)~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~5-0~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~
	~~6-0~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~
	~~7-0~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~
	~~8-8~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~	~~#8 at 48″ o.c.~~
~~9-4~~	~~4-0 (or less)~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~5-0~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~
	~~6-0~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~
	~~7-0~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~
	~~8-0~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~	~~#8 at 48″ o.c.~~
	~~9-4~~	~~#7 at 48″ o.c.~~	~~#8 at 48″ o.c.~~	~~#9 at 48″ o.c.~~
~~10-0~~	~~4-0 (or less)~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~
	~~5-0~~	~~#4 at 48″ o.c.~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~
	~~6-0~~	~~#4 at 48″ o.c.~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~
	~~7-0~~	~~#5 at 48″ o.c.~~	~~#6 at 48″ o.c.~~	~~#7 at 48″ o.c.~~

	8-0	#6 at 48″ o.c.	#7 at 48″ o.c.	#8 at 48″ o.c.
	9-0	#7 at 48″ o.c.	#8 at 48″ o.c.	#9 at 48″ o.c.
	10-0	#7 at 48″ o.c.	#9 at 48″ o.c.	#9 at 48″ o.c.

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.~~

~~a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

~~b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.2.~~

~~c. For alternative reinforcement, see Section 1805.5.3.~~

~~d. For height of unbalanced backfill, see Section 1805.5.1.2.~~

~~TABLE 1805.5(3) - 10-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d^3 6.75 INCHES[a, b, c]~~

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT Soil classes and lateral soil load[a] (psf per foot below natural grade)		
		GW, GP, SW and SP soils 30	GM, GC, SM, SM-SC and ML soils 45	SC, ML-CL and Inorganic CL soils 60
7-4	4-0 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#5 at 56″ o.c.
	7-4	#4 at 56″ o.c.	#5 at 56″ o.c.	#6 at 56″ o.c.
8-0	4-0 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#5 at 56″ o.c.
	7-0	#4 at 56″ o.c.	#5 at 56″ o.c.	#6 at 56″ o.c.
	8-0	#5 at 56″ o.c.	#6 at 56″ o.c.	#7 at 56″ o.c.
8-8	4-0 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#5 at 56″ o.c.
	7-0	#4 at 56″ o.c.	#5 at 56″ o.c.	#6 at 56″ o.c.
	8-8	#5 at 56″ o.c.	#7 at 56″ o.c.	#8 at 56″ o.c.
9-4	4-0 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6-0	#4 at 56″ o.c.	#5 at 56″ o.c.	#5 at 56″ o.c.
	7-0	#4 at 56″ o.c.	#5 at 56″ o.c.	#6 at 56″ o.c.
	8-0	#5 at 56″ o.c.	#6 at 56″ o.c.	#7 at 56″ o.c.
	9-4	#6 at 56″ o.c.	#7 at 56″ o.c.	#8 at 56″ o.c.
10-0	4-0 (or less)	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	5-0	#4 at 56″ o.c.	#4 at 56″ o.c.	#4 at 56″ o.c.
	6-0	#4 at 56″ o.c.	#5 at 56″ o.c.	#5 at 56″ o.c.
	7-0	#5 at 56″ o.c.	#6 at 56″ o.c.	#7 at 56″ o.c.
	8-0	#5 at 56″ o.c.	#7 at 56″ o.c.	#8 at 56″ o.c.
	9-0	#6 at 56″ o.c.	#7 at 56″ o.c.	#9 at 56″ o.c.
	10-0	#7 at 56″ o.c.	#8 at 56″ o.c.	#9 at 56″ o.c.

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.~~

~~a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

~~b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.2.~~

~~c. For alternative reinforcement, see Section 1805.5.3.~~

~~d. For height of unbalanced fill, see Section 1805.5.1.2.~~

~~TABLE 1805.5(4) - 12-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 8.75 INCHES[a, b, c]~~

~~MAXIMUM WALL HEIGHT (feet-inches)~~	~~MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)~~	~~VERTICAL REINFORCEMENT — Soil classes and lateral soil load[a] (psf per foot below natural grade)~~		
		~~GW, GP, SW and SP soils 30~~	~~GM, GC, SM, SM-SC and ML soils 45~~	~~SC, ML-CL and Inorganic CL soils 60~~
~~7-4~~	~~4-0 (or less)~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~5-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~6-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~
	~~7-4~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~
~~8-0~~	~~4-0 (or less)~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~5-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~6-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~
	~~7-0~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~
	~~8-0~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~	~~#7 at 72″ o.c.~~
~~8-8~~	~~4-0 (or less)~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~5-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~6-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~
	~~7-0~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~
	~~8-8~~	~~#5 at 72″ o.c.~~	~~#7 at 72″ o.c.~~	~~#8 at 72″ o.c.~~
~~9-4~~	~~4-0 (or less)~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~5-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~6-0~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#5 at 72″ o.c.~~
	~~7-0~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~
	~~8-0~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~	~~#7 at 72″ o.c.~~
	~~9-4~~	~~#6 at 72″ o.c.~~	~~#7 at 72″ o.c.~~	~~#8 at 72″ o.c.~~
~~10-0~~	~~4-0 (or less)~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~5-0~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~	~~#4 at 72″ o.c.~~
	~~6-0~~	~~#4 at 72″ o.c.~~	~~#5 at 72″ o.c.~~	~~#5 at 72″ o.c.~~
	~~7-0~~	~~#4 at 72″ o.c.~~	~~#6 at 72″ o.c.~~	~~#6 at 72″ o.c.~~
	~~8-0~~	~~#5 at 72″ o.c.~~	~~#6 at 72″ o.c.~~	~~#7 at 72″ o.c.~~
	~~9-0~~	~~#6 at 72″ o.c.~~	~~#7 at 72″ o.c.~~	~~#8 at 72″ o.c.~~
	~~10-0~~	~~#7 at 72″ o.c.~~	~~#8 at 72″ o.c.~~	~~#9 at 72″ o.c.~~

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.~~

~~a. For design lateral soil loads, see Section 1610. Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

~~b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.2.~~

~~c. For alternative reinforcement, see Section 1805.5.3.~~

~~d. For height of unbalanced backfill, see Section 1805.5.1.2.~~

~~TABLE 1805.5(5) - CONCRETE FOUNDATION WALLS[b, c]~~

MAXIMUM WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)	VERTICAL REINFORCEMENT AND SPACING (inches) Design lateral soil load[a] (psf per foot of depth)								
		30			45			60		
		Minimum wall thickness (inches)								
		7.5	9.5	11.5	7.5	9.5	11.5	7.5	9.5	11.5
5	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
6	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	PC	PC	PC
7	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 48″	PC	PC
	7	PC	PC	PC	#5 at 46″	PC	PC	#6 at 48″	PC	PC
8	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 43″	PC	PC
	7	PC	PC	PC	#5 at 41″	PC	PC	#6 at 43″	PC	PC
	8	#5 at 47″	PC	PC	#6 at 43″	PC	PC	#6 at 32″	#6 at 44″	PC
9	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 39″	PC	PC
	7	PC	PC	PC	#5 at 37″	PC	PC	#6 at 38″	#5 at 37″	PC
	8	#5 at 41″	PC	PC	#6 at 38″	#5 at 37″	PC	#7 at 39″	#6 at 39″	#4 at 48″
	9[d]	#6 at 46″	PC	PC	#7 at 41″	#6 at 41″	PC	#7 at 31″	#7 at 41″	#6 at 39″
10	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 37″	PC	PC
	7	PC	PC	PC	#6 at 48″	PC	PC	#6 at 35″	#6 at 48″	PC
	8	#5 at 38″	PC	PC	#7 at 47″	#6 at 47″	PC	#7 at 35″	#7 at 48″	#6 at 45″
	9[d]	#6 at	#4 at	PC	#7 at	#7 at	#4 at 48″	#6 at	#7 at	#7 at 47″

		~~41″~~	~~48″~~		~~37″~~	~~48″~~		~~22″~~	~~37″~~	
	~~10[d]~~	~~#7 at 45″~~	~~#6 at 45″~~	~~PC~~	~~#7 at 31″~~	~~#7 at 40″~~	~~#6 at 38″~~	~~#6 at 22″~~	~~#7 at 30″~~	~~#7 at 38″~~

~~For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot = 0.157 kPa/m.~~

~~a. For design lateral soil loads for different classes of soil, see Section 1610.~~

~~b. Provisions for this table are based on construction requirements specified in Section 1805.5.2.1.~~

~~c. "PC" means plain concrete.~~

~~d. Where design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.~~

~~e. For height of unbalanced backfill, see Section 1805.5.1.2.~~

~~**1805.5.1 Foundation wall thickness.** The minimum thickness of concrete and masonry foundation walls shall comply with Sections 1805.5.1.1 through 1805.5.1.3.~~

~~**1805.5.1.1 Thickness at top of foundation wall.** The thickness of foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8-inch (203 mm) nominal width are permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section 1805.5.1.2 are met. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm); the hollow space behind the corbelled masonry shall be filled with mortar or grout.~~

~~**1805.5.1.2 Thickness based on soil loads, unbalanced backfill height and wall height.** The thickness of foundation walls shall comply with the requirements of Table 1805.5(5) for concrete walls, Table 1805.5(1) for plain masonry walls or Table 1805.5(2), 1805.5(3) or 1805.5(4) for masonry walls with reinforcement. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.~~

~~Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height is permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.~~

~~**1805.5.1.3 Rubble stone.** Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundations for structures in Seismic Design Category C, D, E or F.~~

~~**1805.5.2 Foundation wall materials.** Concrete foundation walls constructed in accordance with Table 1805.5(5) shall comply with Section 1805.5.2.1. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4) shall comply with Section 1805.5.2.2.~~

~~**1805.5.2.1 Concrete foundation walls.** Concrete foundation walls shall comply with the following:~~

1. ~~The size and spacing of vertical reinforcement shown in Table 1805.5(5) is based on the use of reinforcement with a minimum yield strength of 60,000 psi (414 MPa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 MPa) or 50,000 psi (345 MPa) is permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.67 or 0.83, respectively.~~

2. ~~Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, d, from the outside face (soil side) of the wall. The distance, d, is equal to the wall thickness, t, minus 1.25 inches (32 mm) plus one-half the bar diameter, db [d = t - (1.25 + db/2)]. The reinforcement shall~~

~~be placed within a tolerance of ± 3/8 inch (9.5 mm) where d is less than or equal to 8 inches (203 mm) or ± 1/2 inch (2.7 mm) where d is greater that 8 inches (203 mm).~~

~~3. In lieu of the reinforcement shown in Table 1805.5(5), smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length of wall are permitted.~~

~~4. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 3/4 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1.5 inches (38 mm) for No. 5 bars and smaller and not less than 2 inches (51 mm) for larger bars.~~

~~5. Concrete shall have a specified compressive strength ,fc′, of not less than 2,500 psi (17.2 MPa) at 28 days.~~

~~6. The unfactored axial load per linear foot of wall shall not exceed 1.2 tfc′, where t is the specified wall thickness in inches.~~

~~**1805.5.2.2 Masonry foundation walls.** Masonry foundation walls shall comply with the following:~~

~~1. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 MPa).~~

~~2. The specified location of the reinforcement shall equal or exceed the effective depth distance ,d, noted in Tables 1805.5(2), 1805.5(3) and 1805.5(4) and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ASCE 6/TMS 402, Article 3.4 B7 of the specified location.~~

~~3. Grout shall comply with Section 2103.12.~~

~~4. Concrete masonry units shall comply with ASTM C 90.~~

~~5. Clay masonry units shall comply with ASTM C 652 for hollow brick, except compliance with ASTM C 62 or ASTM C 216 is permitted when solid masonry units are installed in accordance with Table 1805.5(1) for plain masonry.~~

~~6. Masonry units shall be installed with Type M or S mortar in accordance with Section 2103.8.~~

~~7. The unfactored axial load per linear foot of wall shall not exceed 1.2tf ′m where t is the specified well thickness in inches and f ′m is the specified compressive strength of masonry in pounds per square inch.~~

~~**1805.5.3 Alternative foundation wall reinforcement.** In lieu of the reinforcement provisions for masonry foundation walls in Table 1805.5(2), 1805.5(3) or 1805.5(4), alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall are permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.~~

~~**1805.5.4 Hollow masonry walls.** At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.~~

~~**1805.5.5 Seismic requirements.** Tables 1805.5(1) through 1805.5(5) shall be subject to the following limitations in Sections 1805.5.5.1 and 1805.5.5.2 based on the seismic design category assigned to the structure as defined in Section 1613.~~

~~**1805.5.5.1 Seismic requirements for concrete foundation walls.** Concrete foundation walls designed using Table 1805.5(5) shall be subject to the following limitations:~~

~~1. Seismic Design Categories A and B. No additional seismic requirements, except provide not less than two No. 5 bars around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of the openings.~~

~~2. Seismic Design Categories C, D, E and F. Tables shall not be used except as allowed for plain concrete members in Section 1908.1.15.~~

~~**1805.5.5.2 Seismic requirements for masonry foundation walls.** Masonry foundation walls designed using Tables 1805.5(1) through 1805.5(4) shall be subject to the following limitations:~~

~~1. Seismic Design Categories A and B. No additional seismic requirements.~~

~~2. Seismic Design Category C. A design using Tables 1805.5(1) through 1805.5(4) is subject to the seismic requirements of Section 2106.4.~~

~~3. Seismic Design Category D. A design using Tables 1805.2(2) through 1805.5(4) is subject to the seismic requirements of Section 2106.5.~~

~~4. Seismic Design Categories E and F. A design using Tables 1805.2(2) through 1805.5(4) is subject to the seismic requirements of Section 2106.6.~~

~~1805A.5.6~~ ***1805A.5.1*** **Foundation wall drainage.** Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807*A*.4.2 and 1807*A*.4.3.

~~1805A.5.7~~ ***1805A.5.2*** **Pier and curtain wall foundations.** *Not permitted by OSHPD and DSA-SS.* ~~Except in Seismic Design Categories D, E and F, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories in height, provided the following requirements are met:~~

~~1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.~~

~~2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3.625 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).~~

~~3. Piers shall be constructed in accordance with Chapter 21 and the following:~~

~~3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.~~

~~3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.~~

~~**Exception:** Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.~~

~~3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.~~

~~4. The maximum height of a 4-inch (102 mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.~~

~~5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305 mm) for hollow masonry.~~

1805*A*.6 Foundation plate or sill bolting. Wood foundation plates or sills shall be bolted or strapped to the foundation or foundation wall as provided in Chapter 23. *Cold formed steel stud foundation plates or sills shall be bolted or fastened to the foundation or foundation wall as provided in Section 2210A.4.*

1805*A*.7 Designs employing lateral bearing. Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall conform to the requirements of Sections 1805*A*.7.1 through 1805*A*.7.3.

1805*A*.7.1 Limitations. The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.

2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

Wood poles shall be treated in accordance with AWPA U1 for sawn timber posts (Commodity Specification A, Use Category 4B) and for round timber posts (Commodity Specification B, Use Category 4B).

1805*A*.7.2 Design criteria. The depth to resist lateral loads shall be determined by the design criteria established in Sections 1805*A*.7.2.1 through 1805*A*.7.2.3, or by other methods approved by the building official.

1805*A*.7.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as a structural diaphragm.

$d = 0.5A\{1 + [1 + (4.36h/A)]^{1/2}\}$ **(Equation 18*A*-1)**

where:

$A = 2.34P/S_1\ b$.

b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.

h = Distance in feet (m) from ground surface to point of application of "P."

P = Applied lateral force in pounds (kN).

S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1804*A*.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

1805*A*.7.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$d^2 = 4.25(Ph/S_3\ b)$ **(Equation 18*A*-2)**

or alternatively

$d^2 = 4.25\ (M_g/S_3 b)$ **(Equation 18*A*-3)**

where:

M_g = Moment in the post at grade, in foot-pounds (kN-m).

S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1804*A*.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

1805*A*.7.2.3 Vertical load. The resistance to vertical loads shall be determined by the allowable soil-bearing pressure set forth in Table 1804*A*.2.

1805*A*.7.3 Backfill. The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 psi (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.

2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.

3. Backfill shall be of controlled low-strength material (CLSM).

1805*A*.8 Design for expansive soils. Footings or foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section 1805*A*.8.1 or 1805*A*.8.2.

Footing or foundation design need not comply with Section 1805*A*.8.1 or 1805*A*.8.2 where the soil is removed in accordance with Section 1805*A*.8.3, nor where the building official approves stabilization of the soil in accordance with Section 1805*A*.8.4.

1805*A*.8.1 Foundations. Footings or foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.

2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

1805*A*.8.2 Slab-on-ground foundations. Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*. Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

1805*A*.8.3 Removal of expansive soil. Where expansive soil is removed in lieu of designing footings or foundations in accordance with Section 1805*A*.8.1 or 1805*A*.8.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 1803*A*.5 or 1803*A*.6.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

1805*A*.8.4 Stabilization. Where the active zone of expansive soils is stabilized in lieu of designing footings or foundations in accordance with Section 1805*A*.8.1 or 1805*A*.8.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

1805*A*.9 Seismic requirements. See Section 1908*A* for additional requirements for footings and foundations of structures assigned to Seismic Design Category ~~C,~~ D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 to 21.10.3, shall apply when not in conflict with the provisions of Section 1805*A*. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.

2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Sections 21.10.1 to 21.10.3.

SECTION 1806*A* - RETAINING WALLS *AND CANTILEVER WALLS*

1806*A*.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning.

(Relocated from 1611A.6, CBC 2001) Retaining walls higher than 12 feet (3658 mm), as measured from the top of the foundation, shall be designed to resist the additional earth pressure caused by seismic ground shaking.

The resultant of the vertical loads and lateral pressures using load combination of section 1605A.3 acting on the wall and its base shall pass through the middle half of the bottom of the footing.

Retaining walls shall be restrained against sliding by friction of the base against the earth, by passive resistance of the soil or by a combination of the two. When used, keys may be assumed to lower the plane of frictional resistance and depth of passive resistance to the level of the bottom of the key. Passive resistance pressures shall be assumed to act on a vertical plane located at the toe of the footing. Overturning shall be computed about the bottom of the spread footing. Passive resistance on the face of the wall may be included in computing resistance to overturning. Frictional resistance on the face of the wall may be included in computing resistance to overturning, except when lateral loads include seismic forces. ~~*See Section 1611A.13 for overturning provisions for free standing walls.*~~

Gravity-type retaining walls utilizing precast concrete units may be used as an alternative to the conventional cantilever retaining systems only after they have been accepted by the enforcement agency.

1806A.2 *(Relocated from 1611A.13, CBC 2001)* ***Freestanding Cantilever Walls.*** *A stability check against the possibility of overturning shall be performed for isolated spread footings which support freestanding cantilever walls. The stability check shall be made by* ~~*multiplying the lateral forces by two*~~ *dividing R_p used for the wall by 2.0. The allowable soil pressure may be doubled for this evaluation.*

> ***EXCEPTION:*** *For overturning about the principal axis of rectangular footings with symmetrical vertical loading and the design lateral force applied, a triangular or trapezoidal soil pressure distribution which covers the full width of the footing will meet the stability requirement.*

SECTION 1807*A* - DAMPPROOFING AND WATERPROOFING

1807*A*.1 Where required. Walls or portions thereof that retain earth and enclose interior spaces and floors below grade shall be waterproofed and dampproofed in accordance with this section, with the exception of those spaces containing groups other than residential and institutional where such omission is not detrimental to the building or occupancy.

Ventilation for crawl spaces shall comply with Section 1203.4.

1807*A*.1.1 Story above grade. Where a basement is considered a story above grade and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be dampproofed in accordance with Section 1807*A*.2 and a foundation drain shall be installed in accordance with Section 1807*A*.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections 1802*A*.2.3, 1807*A*.3 and 1807*A*.4.1 shall not apply in this case.

1807*A*.1.2 Under-floor space. The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the ground-water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections 1802*A*.2.3, 1807*A*.2, 1807*A*.3 and 1807*A*.4 shall not apply in this case.

1807*A*.1.2.1 Flood hazard areas. For buildings and structures in flood hazard areas as established in Section 1612*A*.3, the finished ground level of an under-floor space such as a crawl space shall be equal to or higher than the outside finished ground level.

Exception: Under-floor spaces of Group R-3 buildings that meet the requirements of FEMA/ FIA-TB-11.

1807*A*.1.3 Ground-water control. Where the ground-water table is lowered and maintained at an elevation not less than 6 inches (152 mm) below the bottom of the lowest floor, the floor and walls shall be dampproofed in accordance with Section 1807*A*.2. The design of the system to lower the ground-water table shall be based on accepted principles of engineering that shall consider, but not necessarily be limited to, permeability of the soil, rate at which water enters the drainage system, rated capacity of pumps, head against which pumps are to operate and the rated capacity of the disposal area of the system.

1807*A*.2 Dampproofing required. Where hydrostatic pressure will not occur as determined by Section 1802*A*.2.3, floors and walls ~~for other than wood foundation systems~~ shall be dampproofed in accordance with this section. ~~Wood foundation systems shall be constructed in accordance with AF&PA Technical Report No. 7.~~

1807*A*.2.1 Floors. Dampproofing materials for floors shall be installed between the floor and the base course required by Section 1807*A*.4.1, except where a separate floor is provided above a concrete slab.

Where installed beneath the slab, dampproofing shall consist of not less than 6-mil (0.006 inch; 0.152 mm) polyethylene with joints lapped not less than 6 inches (152 mm), or other approved methods or materials. Where permitted to be installed on top of the slab, dampproofing shall consist of mopped-on bitumen, not less than 4-mil (0.004 inch; 0.102 mm) polyethylene, or other approved methods or materials. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1807*A*.2.2 Walls. Dampproofing materials for walls shall be installed on the exterior surface of the wall, and shall extend from the top of the footing to above ground level.

Dampproofing shall consist of a bituminous material, 3 pounds per square yard (16 N/m^2) of acrylic modified cement, 0.125 inch (3.2 mm) coat of surface-bonding mortar complying with ASTM C 887, any of the materials permitted for waterproofing by Section 1807*A*.3.2 or other approved methods or materials.

1807*A*.2.2.1 Surface preparation of walls. Prior to application of dampproofing materials on concrete walls, holes and recesses resulting from the removal of form ties shall be sealed with a bituminous material or other approved methods or materials. Unit masonry walls shall be parged on the exterior surface below ground level with not less than 0.375 inch (9.5 mm) of portland cement mortar. The parging shall be coved at the footing.

Exception: Parging of unit masonry walls is not required where a material is approved for direct application to the masonry.

1807*A*.3 Waterproofing required. Where the ground-water investigation required by Section 1802*A*.2.3 indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section 1807*A*.1.3, walls and floors shall be waterproofed in accordance with this section.

1807*A*.3.1 Floors. Floors required to be waterproofed shall be of concrete and designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, fully adhered / fully bonded HDPE or polyolefin composite membrane or not less than 6-mil [0.006 inch (0.152 mm)] polyvinyl chloride with joints lapped not less than 6 inches (152 mm) or other approved materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1807*A*.3.2 Walls. Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.

Waterproofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be dampproofed in accordance with Section 1807*A*.2.2. Waterproofing shall consist of two-ply hot-mopped felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1807*A*.3.2.1 Surface preparation of walls. Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 1807*A*.2.2.1.

1807*A*.3.3 Joints and penetrations. Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water-tight utilizing approved methods and materials.

1807*A*.4 Subsoil drainage system. Where a hydrostatic pressure condition does not exist, dampproofing shall be provided and a base shall be installed under the floor and a drain installed around the foundation perimeter. A subsoil drainage system designed and constructed in accordance with Section 1807*A*.1.3 shall be deemed adequate for lowering the ground-water table.

1807*A*.4.1 Floor base course. Floors of basements, except as provided for in Section 1807*A*.1.1, shall be placed over a floor base course not less than 4 inches (102 mm) in thickness that consists of gravel or crushed stone containing not more than 10 percent of material that passes through a No. 4 (4.75 mm) sieve.

Exception: Where a site is located in well-drained gravel or sand/gravel mixture soils, a floor base course is not required.

1807*A*.4.2 Foundation drain. A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an approved filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an approved filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section 1807*A*.4.1, and shall be covered with not less than 6 inches (152 mm) of the same material.

1807*A*.4.3 Drainage discharge. The floor base and foundation perimeter drain shall discharge by gravity or mechanical means into an approved drainage system that complies with the ~~*International*~~ California *Plumbing Code*.

Exception: Where a site is located in well-drained gravel or sand / gravel mixture soils, a dedicated drainage system is not required.

SECTION 1808*A* - PIER AND PILE FOUNDATIONS

1808*A*.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

FLEXURAL LENGTH. Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

MICROPILES. Micropiles are 12-inch-diameter (305 mm) or less bored, grouted-in-place piles incorporating steel pipe (casing) and/or steel reinforcement.

PIER FOUNDATIONS. Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.

Belled piers. Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.

PILE FOUNDATIONS. Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, end bearing or a combination of both.

Augered uncased piles. Augered uncased piles are constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.

Caisson piles. Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.

Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.

Driven uncased piles. Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.

Enlarged base piles. Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.

Steel-cased piles. Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.

Timber piles. Timber piles are round, tapered timbers with the small (tip) end embedded into the soil.

1808*A*.2 Piers and piles—general requirements.

1808*A*.2.1 Design. Piles are permitted to be designed in accordance with provisions for piers in Section 1808*A* and Sections 1812*A*.3 through 1812*A*.10 where either of the following conditions exists, subject to the approval of the building official:

1. Group R-3 and U occupancies not exceeding two stories of light-frame construction, or

2. Where the surrounding foundation materials furnish adequate lateral support for the pile.

1808*A*.2.2 General. Pier and pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802*A*, unless sufficient data upon which to base the design and installation is available.

The investigation and report provisions of Section 1802*A* shall be expanded to include, but not be limited to, the following:

1. Recommended pier or pile types and installed capacities.

2. Recommended center-to-center spacing of piers or piles.

3. Driving criteria.

4. Installation procedures.

5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).

6. Pier or pile load test requirements.

7. Durability of pier or pile materials.

8. Designation of bearing stratum or strata.

9. Reductions for group action, where necessary.

1808*A*.2.3 Special types of piles. The use of types of piles not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such piles. The allowable stresses shall not in any case exceed the limitations specified herein.

1808*A*.2.4 Pile caps. Pile caps shall be of reinforced concrete, and shall include all elements to which piles are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of piles shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of piles. The tops of piles shall be cut back to sound material before capping.

1808*A*.2.5 Stability. Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official.

Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories or 35 feet (10 668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

1808*A*.2.6 Structural integrity. Piers or piles shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of piles being installed or already in place.

1808*A*.2.7 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the pier or pile during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Splices shall develop not less than 50 percent of the least capacity of the pier or pile in bending. In addition, splices occurring in the upper 10 feet (3048 mm) of the embedded portion of the pier or pile shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the pier or pile load of 3 inches (76 mm), or the pier or pile shall be braced in accordance with Section 1808*A*.2.5 to other piers or piles that do not have splices in the upper 10 feet (3048 mm) of embedment.

1808*A*.2.8 Allowable pier or pile loads.

1808*A*.2.8.1 Determination of allowable loads. The allowable axial and lateral loads on piers or piles shall be determined by an approved formula, load tests or method of analysis.

1808*A*.2.8.2 Driving criteria. The allowable compressive load on any pile where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate pile driveability of both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section 1808*A*.2.8.3. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven piles. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

1808*A*.2.8.3 Load tests. Where design compressive loads per pier or pile are greater than those permitted by Section 1808*A*.2.10 or where the design load for any pier or pile foundation is in doubt, control test piers or piles shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one pier or pile shall be test loaded in each area of uniform subsoil conditions. Where required by the building official, additional piers or piles shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test pier or pile as assessed by one of the published methods listed in Section 1808*A*.2.8.3.1 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section 1808*A*.2.12. In subsequent installation of the balance of foundation piles, all piles shall be deemed to have a supporting capacity equal to the control pile where such piles are of the same type, size and relative length as the test pile; are installed using the same or comparable methods and equipment as the test pile; are installed in similar subsoil conditions as the test pile; and, for driven piles, where the rate of penetration (e.g., net displacement per blow) of such piles is equal to or less than that of the test pile driven with the same hammer through a comparable driving distance.

1808*A*.2.8.3.1 Load test evaluation. It shall be permitted to evaluate pile load tests with any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90% Criterion.
3. Butler-Hoy Criterion.
4. Other methods approved by the building official.

1808*A*.2.8.4 Allowable frictional resistance. The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804*A*.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a soil investigation, as specified in Section 1802*A*, is submitted or a greater value is substantiated by a load test in accordance with Section 1808*A*.2.8.3. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802*A*.

1808*A*.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808*A*.2.8.3 divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

1808*A*.2.8.6 Load-bearing capacity. Piers, individual piles and groups of piles shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

1808*A*.2.8.7 Bent piers or piles. The load-bearing capacity of piers or piles discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative pier or pile.

1808*A*.2.8.8 Overloads on piers or piles. The maximum compressive load on any pier or pile due to mislocation shall not exceed 110 percent of the allowable design load.

1808*A*.2.9 Lateral support.

1808*A*.2.9.1 General. Any soil other than fluid soil shall be deemed to afford sufficient lateral support to the pier or pile to prevent buckling and to permit the design of the pier or pile in accordance with accepted engineering practice and the applicable provisions of this code.

1808*A*.2.9.2 Unbraced piles. Piles standing unbraced in air, water or in fluid soils shall be designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered fixed and laterally supported at 5 feet (1524 mm) below the ground surface and in soft material at 10 feet (3048 mm) below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

1808*A*.2.9.3 Allowable lateral load. Where required by the design, the lateral load capacity of a pier, a single pile or a pile group shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25 mm) at the ground surface.

1808*A*.2.10 Use of higher allowable pier or pile stresses. Allowable stresses greater than those specified for piers or for each pile type in Sections 1809*A* and 1810*A* are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802*A*.

2. Pier or pile load tests in accordance with Section 1808*A*.2.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

1808*A*.2.11 Piles in subsiding areas. Where piles are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the piles by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter are permitted to be increased where satisfactory substantiating data are submitted.

1808*A*.2.12 Settlement analysis. The settlement of piers, individual piles or groups of piles shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

1808*A*.2.13 Preexcavation. The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for piers or piles subject to load tests and in such a manner that will not impair the carrying capacity of the piers or piles already in place or damage adjacent structures. Pile tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.

1808*A*.2.14 Installation sequence. Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.

1808*A*.2.15 Use of vibratory drivers. Vibratory drivers shall only be used to install piles where the pile load capacity is verified by load tests in accordance with Section 1808*A*.2.8.3. The installation of production piles shall be controlled according to power consumption, rate of penetration or other approved means that ensure pile capacities equal or exceed those of the test piles.

1808*A*.2.16 Pile driveability. Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.

1808*A*.2.17 Protection of pile materials. Where boring records or site conditions indicate possible deleterious action on pier or pile materials because of soil constituents, changing water levels or other factors, the pier or pile materials shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the piles so as not to be rendered ineffective by driving. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

1808*A*.2.18 Use of existing piers or piles. Piers or piles left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the piers or piles are sound and meet the requirements of this code. Such piers or piles shall be load tested or redriven to verify their capacities. The design load applied to such piers or piles shall be the lowest allowable load as determined by tests or redriving data.

1808*A*.2.19 Heaved piles. Piles that have heaved during the driving of adjacent piles shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the pile shall be verified by load tests in accordance with Section 1808*A*.2.8.3.

1808*A*.2.20 Identification. Pier or pile materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

1808*A*.2.21 Pier or pile location plan. A plan showing the location and designation of piers or piles by an identification system shall be filed with the building official prior to installation of such piers or piles. Detailed records for piers or individual piles shall bear an identification corresponding to that shown on the plan.

1808A.2.22 Special inspection. Special inspections in accordance with Sections 1704*A*.8 and 1704*A*.9 shall be provided for piles and piers, respectively.

1808*A*.2.23 Seismic design of piers or piles.

1808*A*.2.23.1 Seismic Design Category C. Where a structure is assigned to Seismic Design Category C ~~in accordance with Section 1613~~, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade, reinforced concrete slabs on grade, confinement by competent rock, hard cohesive soils or very dense granular soils.

Exception: Piers supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, lightly loaded exterior decks and patios of Group R-3 and U occupancies not exceeding two stories of light-frame construction, are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

1808*A*.2.23.1.1 Connection to pile cap. Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region.

Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

Exception: Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Section 1605*A*.4.

1808*A*.2.23.1.2 Design details. Pier or pile moments, shears and lateral deflections used for design shall be established considering the nonlinear interaction of the shaft and soil, as recommended by a registered design professional. Where the ratio of the depth of embedment of the pile-to-pile diameter or width is less than or equal to six, the pile may be assumed to be rigid.

Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile.

Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.

1808*A*.2.23.2 Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613*A*, the requirements for Seismic Design Category C given in Section 1808*A*.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808*A* through 1812*A*. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less in height are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.

2. *Not permitted by OSHPD and DSA-SS.* ~~Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.~~

3. *Not permitted by OSHPD and DSA-SS.* ~~Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.~~

1808*A*.2.23.2.1 Design details for piers, piles and grade beams. Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Section 1613*A*.5.2, shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Sections 1809*A*.2.3.2.1 and 1809*A*.2.3.2.2 shall apply. Grade beams shall be designed as beams in accordance with ACI 318, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 1605*A*.4, they need not conform to ACI 318, Chapter 21.

1808*A*.2.23.2.2 Connection to pile cap. For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations of Section 1605*A*.4.

2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations of Section 1605*A*.4 or development of the full axial, bending and shear nominal strength of the pile.

1808_A_.2.23.2.3 Flexural strength. Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.

The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations of Section 1605_A_.4.

1808A.2.23.2.4 Deformation *(Relocated from 1806A.8.1, CBC 2001) Piles and ~~caissons~~ piers used to support lateral loads from structures shall be designed with due consideration to the ~~elastic~~ deformation of the piles, ~~caissons~~ piers, pile caps and connecting grade beams.*

SECTION 1809_A_ DRIVEN PILE FOUNDATIONS

1809_A_.1 Timber piles. _Not permitted by OSHPD and DSA-SS._ ~~Timber piles shall be designed in accordance with the AF&PA NDS.~~

~~1809.1.1 Materials. Round timber piles shall conform to ASTM D 25. Sawn timber piles shall conform to DOC PS 20.~~

~~**1809.1.2 Preservative treatment**. Timber piles used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber piles will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber piles and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber piles. Preservative-treated timber piles shall be subject to a quality control program administered by an approved agency. Pile cutoffs shall be treated in accordance with AWPA M4.~~

~~1809.1.3 Defective piles. Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.~~

~~**1809.1.4 Allowable stresses.** The allowable stresses shall be in accordance with the AF&PA NDS.~~

1809_A_.2 Precast concrete piles.

1809_A_.2.1 General. The materials, reinforcement and installation of precast concrete piles shall conform to Sections 1809_A_.2.1.1 through 1809_A_.2.1.4.

1809_A_.2.1.1 Design and manufacture. Piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

1809_A_.2.1.2 Minimum dimension. The minimum lateral dimension shall be 8 inches (203 mm). Corners of square piles shall be chamfered.

1809_A_.2.1.3 Reinforcement. Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced not more than 4 inches (102 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends of the pile; and not more than 6 inches (152 mm) elsewhere except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25 mm) center to center. The gage of ties and spirals shall be as follows:

For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).

For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).

For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than 0.25 inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

1809*A*.2.1.4 Installation. Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.

1809*A*.2.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall conform to Sections 1809*A*.2.2.1 through 1809*A*.2.2.5.

1809*A*.2.2.1 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).

1809*A*.2.2.2 Minimum reinforcement. The minimum amount of longitudinal reinforcement shall be 0.8 percent of the concrete section and shall consist of at least four bars.

1809*A*.2.2.2.1 Seismic reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C ~~in accordance with Section 1613~~, the following shall apply. Longitudinal reinforcement with a minimum steel ratio of 0.01 shall be provided throughout the length of precast concrete piles. Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with closed ties or spirals of a minimum $^3/_8$ inch (9.5 mm) diameter. Ties or spirals shall be provided at a maximum spacing of eight times the diameter of the smallest longitudinal bar, not to exceed 6 inches (152 mm). Throughout the remainder of the pile, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal bar diameter not to exceed 8 inches (203 mm).

1809*A*.2.2.2.2 Seismic reinforcement in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613*A*, the requirements for Seismic Design Category C in Section 1809*A*.2.2.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three pile diameters of the bottom of the pile cap. For other than Site Class E or F, or liquefiable sites and where spirals are used as the transverse reinforcement, a volumetric ratio of spiral reinforcement of not less than one-half that required by Section 21.4.4.1(a) of ACI 318 shall be permitted.

1809*A*.2.2.3 Allowable stresses. The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 24,000 psi (165 MPa).

1809*A*.2.2.4 Installation. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1809*A*.2.2.5 Concrete cover. Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm).

Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.

Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).

1809*A*.2.3 Precast prestressed piles. Precast prestressed concrete piles shall conform to the requirements of Sections 1809*A*.2.3.1 through 1809*A*.2.3.5.

1809*A*.2.3.1 Materials. Prestressing steel shall conform to ASTM A 416. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).

1809*A*.2.3.2 Design. Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and

700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

1809*A*.2.3.2.1 Design in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C ~~in accordance with Section 1613~~, the following shall apply. The minimum volumetric ratio of spiral reinforcement shall not be less than 0.007 or the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$\rho_s = 0.12 f'_c / f_{yh}$ **(Equation 18*A*-4)**

where:

f'_c = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 18A-4 shall be provided below the upper 20 feet (6096 mm) of the pile.

The pile cap connection by means of dowels as indicated in Section 1808*A*.2.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.

1809*A*.2.3.2.2 Design in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613*A*, the requirements for Seismic Design Category C in Section 1809*A*.2.3.2.1 shall be met, in addition to the following:

1. Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.

2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.

3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longtitudinal strand, or 8 inches (203 mm), whichever is smaller.

4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of the spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Sec. 12.14.3 of ACI 318.

5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

   $\rho_s = 0.25(f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$
   (Equation 18*A*-5)

   but not less than:

   $\rho_s = 0.12(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]$
   (Equation 18*A*-6)

   and need not exceed:

   $\rho_s = 0.021$ **(Equation 18*A*-7)**

where:

A_g = Pile cross-sectional area, square inches (mm^2).

A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).

f'_c = Specified compressive strength of concrete, psi (MPa).

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

P = Axial load on pile, pounds (kN), as determined from Equations 16*A*-5 and 16*A*-6.

ρ_s = Volumetric ratio (vol. spiral/ vol. core).

~~This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.~~

6. When transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacings, and perpendicular to dimension ,h_c, shall conform to:

$A_{sh} = 0.3sh_c (f'_c/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$ **(Equation 18*A*-8)**

but not less than:

$A_{sh} = 0.12sh_c (f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)]$ **(Equation 18*A*-9)**

where:

$f_{yh} = \leq 70{,}000$ psi (483 MPa).

h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).

s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross-sectional area of transverse reinforcement, square inches (mm^2).

f'_c = Specified compressive strength of concrete, psi (MPa).

The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

1809*A*.2.3.3 Allowable stresses. The allowable design compressive stress, f_c, in concrete shall be determined as follows:

$f_c = 0.33 f'_c - 0.27 f_{pc}$ **(Equation 18*A*-10)**

where:

f'_c = The 28-day specified compressive strength of the concrete.

f_{pc} = The effective prestress stress on the gross section.

1809*A*.2.3.4 Installation. A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1809*A*.2.3.5 Concrete cover. Prestressing steel and pile reinforcement shall have a concrete cover of not less than $1^1/_4$ inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and $1^1/_2$ inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than $2^1/_2$ inches (64 mm).

1809*A*.3 Structural steel piles. Structural steel piles shall conform to the requirements of Sections 1809*A*.3.1 through 1809*A*.3.4.

1809*A*.3.1 Materials. Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.

1809*A*.3.2 Allowable stresses. The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (F_y).

Exception: Where justified in accordance with Section 1808*A*.2.10, the allowable axial stress is permitted to be increased above $0.35F_y$, but shall not exceed $0.5F_y$.

1809*A*.3.3 Dimensions of H-piles. Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.

2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).

3. Flanges and web shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

1809*A*.3.4 Dimensions of steel pipe piles. Steel pipe piles driven open ended shall have a nominal outside diameter of not less than 8 inches (203 mm). The pipe shall have a minimum cross section of 0.34 square inch (219 mm^2) to resist each 1,000 foot-pounds (1356 N-m) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 Mpa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where pipe wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided.

SECTION 1810*A* - CAST-IN-PLACE CONCRETE PILE FOUNDATIONS

1810*A*.1 General. The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 1810*A*.1.1 through 1810*A*.1.3.

1810*A*.1.1 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pile, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

1810*A*.1.2 Reinforcement. Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 1810*A*.3.4, reinforcement where required shall be assembled and tied together and shall be placed in the pile as a unit before the reinforced portion of the pile is filled with concrete except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state.

1810*A*.1.2.1 Reinforcement in Seismic Design Category C. Where a structure is assigned to Seismic Design Category C ~~in accordance with Section 1613~~, the following shall apply. A minimum longitudinal reinforcement ratio of 0.0025 shall be provided for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top one-third of the pile length, a minimum length of 10 feet (3048 mm) below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with closed ties (or equivalent spirals) of a minimum $^3/_8$ inch (9 mm) diameter provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcement with a maximum spacing of 6 inches (152 mm) or 8-longitudinal-bar diameters, whichever is less, shall be provided within a distance equal to three times the least pile dimension of the bottom of the pile cap.

1810*A*.1.2.2 Reinforcement in Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613*A*, the requirements for Seismic Design Category C given above shall be met, in addition to the following. A minimum longitudinal reinforcement ratio of 0.005 shall be provided for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top one-half of the pile length a minimum length of 10 feet (3048 mm) below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least pile dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 for other than Class E, F or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall neither exceed 12-longitudinal-bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of No. 3 bars for piles with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger piles.

1810*A*.1.3 Concrete placement. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.

1810*A*.2 Enlarged base piles. Enlarged base piles shall conform to the requirements of Sections 1810*A*.2.1 through 1810*A*.2.5. *Enlarged base piles shall be considered as an alternative system.*

1810*A*.2.1 Materials. The maximum size for coarse aggregate for concrete shall be $^3/_4$ inch (19.1 mm). Concrete to be compacted shall have a zero slump.

1810*A*.2.2 Allowable stresses. The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (f'_c). Where the concrete is place in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (f'_c).

1810*A*.2.3 Installation. Enlarged bases formed either by compacting concrete or driving a precast base shall be formed in or driven into granular soils. Piles shall be constructed in the same manner as successful prototype test piles driven for the project. Pile shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the pile shaft shall be filled sufficiently to reestablish lateral support by the soil. Where pile heave occurs, the pile shall be replaced unless it is demonstrated that the pile is undamaged and capable of carrying twice its design load.

1810*A*.2.4 Load-bearing capacity. Pile load-bearing capacity shall be verified by load tests in accordance with Section 1808*A*.2.8.3.

1810*A*.2.5 Concrete cover. The minimum concrete cover shall be $2^1/_2$inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.

1810*A*.3 Drilled or augered uncased piles. Drilled or augered uncased piles shall conform to Sections 1810*A*.3.1 through 1810*A*.3.5.

1810*A*.3.1 Allowable stresses. The allowable design stress in the concrete of drilled or augered uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).

1810*A*.3.2 Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved construction documents.

1810*A*.3.3 Installation. Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.

Where concrete is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place piles shall not be installed within six pile diameters center to center of a pile filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed pile drops due to installation of an adjacent pile, the pile shall be replaced.

1810*A*.3.4 Reinforcement. For piles installed with a hollow-stem auger where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete. All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

1810*A*.3.5 Reinforcement in Seismic Design Category ~~C,~~ D, E or F. Where a structure is assigned to Seismic Design Category ~~C,~~ D, E or F in accordance with Section 1613*A*, the corresponding requirements of Sections 1810*A*.1.2.1 and 1810*A*.1.2.2 shall be met.

1810*A*.4 Driven uncased piles. Driven uncased piles shall conform to Sections 1810*A*.4.1 through 1810*A*.4.4.

1810*A*.4.1 Allowable stresses. The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f'_c) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.

1810*A*.4.2 Dimensions. The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).

Exception: The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.

1810*A*.4.3 Installation. Piles shall not be driven within six pile diameters center to center in granular soils or within one-half the pile length in cohesive soils of a pile filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed pile rises or drops, the pile shall be replaced. Piles shall not be installed in soils that could cause pile heave.

1810*A*.4.4 Concrete cover. Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.

1810*A*.5 Steel-cased piles. Steel-cased piles shall comply with the requirements of Sections 1810*A*.5.1 through 1810*A*.5.4.

1810*A*.5.1 Materials. Pile shells or casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

1810*A*.5.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the conditions specified in Sections 1810*A*.5.2.1 through 1810*A*.5.2.4.

1810*A*.5.2.1 Shell thickness. The thickness of the steel shell shall not be less than manufacturer's standard gage No. 14 gage (0.068 inch) (1.75 mm) minimum.

1810*A*.5.2.2 Shell type. The shell shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.

1810*A*.5.2.3 Strength. The ratio of steel yield strength (f_y) to 28-day specified compressive strength (f'_c) shall not be less than six.

1810*A*.5.2.4 Diameter. The nominal pile diameter shall not be greater than 16 inches (406 mm).

1810*A*.5.3 Installation. Steel shells shall be mandrel driven their full length in contact with the surrounding soil.

The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile shall not be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in steel shells within heave range of driving.

1810*A*.5.4 Reinforcement. Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

1810*A*.5.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category ~~C,~~ D, E or F in accordance with Section 1613*A*, the reinforcement requirements for drilled or augered uncased piles in Section 1810*A*.3.5 shall be met.

Exception: A spiral-welded metal casing of a thickness no less than the manufacturer's standard gage No. 14 gage [0.068 inch (1.7 mm)] is permitted to provide concrete confinement in lieu of the closed ties or equivalent spirals required in an uncased concrete pile. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

1810*A*.6 Concrete-filled steel pipe and tube piles. Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 1810*A*.6.1 through 1810*A*.6.5.

1810*A*.6.1 Materials. Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 1810*A*.1.1. The maximum coarse aggregate size shall be $^3/_4$inch (19.1 mm).

1810*A*.6.2 Allowable stresses. The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.

Exception: Where justified in accordance with Section 1808*A*.2.10, the allowable stresses are permitted to be increased to 0.50 F_y.

1810*A*.6.3 Minimum dimensions. Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 1809*A*.3.4. For mandrel-driven pipe piles, the minimum wall thickness shall be $^{1}/_{10}$ inch (2.5 mm).

1810*A*.6.4 Reinforcement. Reinforcement steel shall conform to Section 1810*A*.1.2. Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.

1810*A*.6.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category ~~C,~~ D, E or F in accordance with Section 1613*A*, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than $^{3}/_{16}$inch (5 mm).

1810*A*.6.5 Placing concrete. The placement of concrete shall conform to Section 1810*A*.1.3, but is permitted to be chuted directly into smooth-sided pipes and tubes without a centering funnel hopper.

1810*A*.7 Caisson piles. Caisson piles shall conform to the requirements of Sections 1810*A*.7.1 through 1810*A*.7.6.

1810*A*.7.1 Construction. Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete. The caisson pile shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

1810*A*.7.2 Materials. Pipe and steel cores shall conform to the material requirements in Section 1809*A*.3. Pipes shall have a minimum wall thickness of $^{3}/_{8}$inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).

1810*A*.7.3 Design. The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.

1810*A*.7.4 Structural core. The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

1810*A*.7.5 Allowable stresses. The allowable design compressive stresses shall not exceed the following: concrete, $0.33 f'_c$; steel pipe, $0.35\ F_y$ and structural steel core, $0.50\ F_y$.

1810*A*.7.6 Installation. The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. Concrete shall not be placed through water except where a tremie or other approved method is used.

1810*A*.8 Micropiles. Micropiles shall conform to the requirements of Sections 1810*A*.8.1 through 1810*A*.8.5.

1810*A*.8.1 Construction. Micropiles shall consist of a grouted section reinforced with steel pipe or steel reinforcing. Micropiles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The full length of the micropile shall contain either a steel pipe or steel reinforcement.

1810*A*.8.2 Materials. Grout shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement steel shall be deformed bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150.

Pipe/casing shall have a minimum wall thickness of $^{3}/_{16}$ inch (4.8 mm) and as required to meet Section 1808*A*.2.7. Pipe/casing shall meet the tensile requirements of ASTM A 252 Grade 3, except the minimum yield strength shall be as used in the design submittal [typically 50,000 psi to 80,000 psi (345 MPa to 552 MPa)] and minimum elongation shall be 15 percent.

1810*A*.8.3 Allowable stresses. The allowable design compressive stress on grout shall not exceed 0.33 f'_c. The allowable design compressive stress on steel pipe and steel reinforcement shall not exceed the lesser of 0.4 F_y, or 32,000 psi (220 MPa). The allowable design tensile stress for steel reinforcement shall not exceed 0.60 F_y. The allowable design tensile stress for the cement grout shall be zero.

1810*A*.8.4 Reinforcement. For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the cement grout enclosed within the pipe is permitted to be included at the allowable stress of the grout.

1810*A*.8.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down 120 percent times the flexural length. The flexural length shall be determined in accordance with Section 1808*A*.1. Where a structure is assigned to Seismic Design Category D, E or F, the pile shall be considered as an alternative system. In accordance with Section 104.11, *Appendix Chapter 1*, the alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.

1810*A*.8.5 Installation. The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile. The following requirements apply to specific installation methods:

1. For piles grouted inside a temporary casing, the reinforcing steel shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed.

2. For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.

3. For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.

4. Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.

5. Piles shall be grouted as soon as possible after drilling is completed.

6. For piles designed with casing full length, the casing must be pulled back to the top of the bond zone and reinserted or some other suitable means shall be employed to verify grout coverage outside the casing.

SECTION 1811*A* - COMPOSITE PILES

1811*A*.1 General. Composite piles shall conform to the requirements of Sections 1811*A*.2 through 1811*A*.5.

1811*A*.2 Design. Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation.

1811*A*.3 Limitation of load. The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.

1811*A*.4 Splices. Splices between concrete and steel ~~or wood~~ sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.

1811*A*.5 Seismic reinforcement. Where a structure is assigned to Seismic Design Category ~~C,~~ D, E or F in accordance with Section 1613*A*, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 1810*A*.1.2.1 and 1810*A*1.2.2 or the steel section shall comply with Section 1810*A*.6.4.1.

SECTION 1812*A* - PIER FOUNDATIONS

1812*A*.1 General. Isolated and multiple piers used as foundations shall conform to the requirements of Sections 1812*A*.2 through 1812*A*.10, as well as the applicable provisions of Section 1808*A*.2.

1812*A*.2 Lateral dimensions and height. The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.

1812*A*.3 Materials. Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.

1812*A*.4 Reinforcement. Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete.

> **Exception:** Reinforcement is permitted to be wet set and the $2^{1}/_{2}$- inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.
>
> Reinforcement shall conform to the requirements of Sections 1810*A*.1.2.1 and 1810*A*.1.2.2.
>
> **Exceptions:**

1. Isolated piers supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than a minimum of one No. 4 bar, without ties or spirals, when detailed so the pier is not subject to lateral loads and the soil is determined to be of adequate stiffness.

2. Isolated piers supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than one No. 4 bar, without ties or spirals, when the lateral load ,E, to the top of the pier does not exceed 200 pounds (890 N) and the soil is determined to be of adequate stiffness.

3. Piers supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction are permitted to be reinforced as required by rational analysis but not less than two No. 4 bars, without ties or spirals, when it can be shown the concrete pier will not rupture when designed for the maximum seismic load ,E_m, and the soil is determined to be of adequate stiffness.

4. Closed ties or spirals where required by Section 1810*A*.1.2.2 are permitted to be limited to the top 3 feet (914 mm) of the piers 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

1812*A*.5 Concrete placement. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.

1812*A*.6 Belled bottoms. Where pier foundations are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

1812*A*.7 Masonry. Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced in accordance with ACI 530/ASCE 5/TMS 402.

1812*A*.8 Concrete. ~~Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318. Where the unsupported height to least lateral dimension exceeds three, piers~~ *Piers* shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318.

> **Exception:** *Not permitted by OSHPD and DSA-SS*. ~~Where adequate lateral support is furnished by the surrounding materials as defined in Section 1808.2.9, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318 for bearing on concrete shall apply.~~

1812*A*.9 Steel shell. Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1808*A*.2.17. Horizontal joints in the shell shall be spliced to comply with Section 1808*A*.2.7.

1812*A*.10 Dewatering. Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.

Notation [For DSA-SS]:
Authority: Education Code Sections 17310, 81142; Health & Safety Code Section 16022
Reference(s): Education Code Sections 17280 - 17317, and 811130 - 81149; Health & Safety Code Sections 16000 – 16023

Notation [For OSHPD]:
Authority: Health and Safety Code Section 129850
Reference: Health and Safety Code Sections 1275, 129850 and 129790